AN EXPERIMENTAL STUDY ON STRENGTH BEHAVIOUR OF JOINTED ROCK MASS THROUGH MODELLING UNDER UNIAXIAL COMPRESSION

A PAPER SUBMITTED IN PARTIAL FULFILMENT OF THE REQUIREMENTS FOR THE DEGREE OF

Bachelor of Technology
In
Civil Engineering

BY

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Department of Civil Engineering

National Institute of Technology

Rourkela

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Under the Guidance of
Prof. (Dr.) N. Roy

Department of Civil Engineering

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CERTIFICATE

This is to certify that the thesis entitled "An experimental study on strength behaviour of Jointed Rock Mass through modelling under uniaxial compression tests" submitted by P.Teja in partial fulfilment of the requirements for the award of Bachelor of Technology Degree in Civil Engineering at National Institute of Technology, Rourkela is an authentic work carried out by her under my guidance.

To the best of my knowledge the matter embodied in the thesis has not been submitted to any university/institute for the award of any degree or diploma.

Date:

Prof. N. Roy
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CONTENTS

SYMBOLS USED:

J_f = Joint factor

J_n = Number of joints per metre length.

n = Joint inclination parameter

r = Roughness parameter.

β = Orientation of joint.

i = Inclination of the asperity

σ_{cj} = Uniaxial compressive strength of jointed rock.

σ_{ci} = Uniaxial compressive strength of intact rock

σ_{cr} = Uniaxial compressive ration.

E_j = Tangent modulus of jointed rock

E_i = Tangent modulus of intact rock

E_r = Elastic modulus ratio.

ح = Shear strength

Φ = angle of friction

TABLES USED:

FIGURES USED:

1. INTRODUCTION

Stability analysis of a rock mass is an important and complicated problem related to the safety of engineering buildings. One of the major tasks of engineering geomechanics is to evaluate the rock mass stability both qualitatively and quantitatively. The ultimate strength and deformation of jointed rock mass are important parameters that designers look for in selecting sites for foundations of civil structures in rocks.

In nature rock is exists as a rock mass. It is a discontinuous medium with fissures, fractures, joints, bedding planes, and faults. These discontinuities may exist with or without gouge material. The strength of rock masses depends on the behaviour of these discontinuities or planes of weakness. The frequency of joints, their orientation with respect to the engineering structures, and the roughness of the joint have a significant importance from the stability point of view. Reliable characterization of the strength and deformation behaviour of jointed rocks is very important for safe design of various types of civil structures such as arch dams, bridge piers, and tunnels. aa

The relation between joint factor, σ_{cr} and σ_{cj} is of paramount importance for such study. The best estimate of the design parameters can only be made through large size field testing of the mass and loading it up to failure. It is, however, extremely difficult, if not impossible, to stress a large volume of jointed mass in the field up to ultimate failure. A better alternative is to get the deformability characteristics by stressing a limited area of the mass up to a certain stress level and then relate the ultimate strength of the mass to the laboratory uniaxial compressive strength (UCS) of the rock material.

Laboratory rock testing is performed to determine the strength and elastic properties of intact specimens and the potential for degradation and disintegration of the rock material. The derived parameters are used in part for the design of rock fills, cut slopes, shallow and deep foundations, tunnels, and the assessment of shore protection materials (rip-rap). Deformation and strength properties of intact specimens aid in evaluating the larger-scale rock mass that is significantly controlled by joints, fissures, and discontinuity features (spacing, roughness, orientation, infilling), water pressures, and ambient geostatic stress state.

2. ROCK

Rock may be defined as a granular, aelotropic, heterogeneous technical substance which occurs naturally and which is composed of grains cemented together by mechanical bond but ultimately by atomic, ionic and molecularly within the grains.

Rock is a discontinuous medium with fissures, fractures, joints, bedding planes, and faults. These discontinuities may exist with or without gouge material. The strength of rock masses depends on the behaviour of these discontinuities or planes of weakness. The frequency of joints, their orientation with respect to the engineering structures, and the roughness of the joint have a significant importance from the stability point of view. Reliable characterization of the strength and deformation behaviour of jointed rocks is very important for safe design of civil structures such as arch dams, bridge piers, and tunnels.

2.1 INTACT ROCK MASS:

An intact rock is considered to be an aggregate of mineral, without any structural defects and also such rocks are treated as isotropic, homogeneous and continuous. Their failures can be classified as brittle which implies a sudden reduction in strength when a limited stress level is exceeded.

Strength of intact rock mass is mainly influenced by the following factors:

- Geological
- Lithological
- Physical
- Mechanical
- Environmental factors

TABLE - 2.1
FACTORS AFFECTING THE STRENGTH OF ROCK

Geological	Geological age, weathering and other alternatives
Lithological	Mineral composition, cementing material, texture and fabric, anisotropy.
Physical	Density/specific gravity, void index, porosity
Mechanical	Specimen preparation, geometry, end contact/ end restraint, type of testing machine, plate of loading.
Environmental	Moisture content, nature of pore fluids, temperature, confining pressure.

The shear strength of jointed rock mass depends on the type and nature of origin of the discontinuity, roughness, depth of weathering, and type of filling material. The strength behaviour of rock mass is governed by both intact rock properties and properties of discontinuities. The shear strength of rock mass depends on several factors like:

- Angle made by the joint with the principal stress direction(β)
- Opening of the joint
- Degree of joint separation
- Number of joints in a given direction
- Joint frequency and joint roughness
- Strength along the joint.

3. JOINT ROCK PROPERTIES

3.1 STRENGTH OF ROCKMASS:

The strength of the rock mass is only a fraction of the strength of the intact strength. The reason for this is that failure in the rock mass is a combination of both intact rock strength and separation or sliding along discontinuities. The latter process usually dominates. Sliding on discontinuities occurs against the cohesion and/or frictional resistance along the discontinuity. The cohesion component is only a very small fraction of the cohesion of the intact rock. An important aspect of rock behavior under the uniaxial condition is the change in behavior from brittle to ductile nature at high confining pressure.

3.2 JOINT ROCK INTENSITY:

The joint intensity is the number of per unit distance normal to the plane of joints in a set. It influences the strength and deformation behaviour of the rock mass significantly, strength of rock decreases as the no. of joints increases.

To understand the strength characteristics of jointed rock mass specimen, arora in 1987 introduced a factor (J_f) defined by the expression as:

$$J_f = J_n / n \times r$$

Where J_n = no. of joints per meter length.

n = joint inclination parameter which is a function of joint orientation.

r = roughness parameter i.e. tan φ_j which depends on the joint condition.

The value of 'n' is obtained by taking the ration of log (strength reduction) at $\beta = 90^o$ to log (strength reduction) at the desired value of β. This inclination factor is independent of joint frequency. The joint strength parameter 'r' is obtained from a shear test along the joint and is given as $r = \tau_j / \sigma_{nj}$, where is τ_j the shear strength along the joint and σ_{nj} is the normal stress on the joint. The values of 'n' and 'r' are given by (Ramamurthy, 1994 and Arora, 1987) based on extensive laboratory testing.

3.3 ORIENTATION OF JOINTS

The orientation of joints is one of the most important parameters which influence the resultant shear stress distribution along with nature and extent of failure zones.

On the basis of Mohr Coulomb equation, Jaegar and Cook (1979) reported the criteria for slip in the single weak plane. They developed the following expression to show the variation of deviator stress ($\sigma_1 - \sigma_3$) necessary to cause the failure with the variation of joint β with σ_3 and φ kept fixed.

$$(\sigma_1 - \sigma_3) = (2c + 2\tan\varphi)/\ \{(1 - \tan\varphi.\cot\beta)\ \sin 2\beta\}$$

TABLE – 3.1
INCLINATION PARAMETERS ACCORDING TO ORIENTATION OF THE JOINT

Orientation of joint β°	Inclination parameter n
0	0.810
10	0.460
20	0.105
30	0.046
40	0.071
50	0.306
60	0.465
70	0.634
80	0.814
90	1.00

3.4 JOINT ROUGHNESS:

Joint roughness is of utmost importance to the shear behaviour of joints. This is because joint roughness has a fundamental influence on the development of dilation and as a consequence the strength of joint during relative shear displacement. When a fractured rock surface is viewed under a magnification the profile exhibits a random arrangement of peaks and valleys called asperities forming a rough surface. The surface roughness is due to asperities with short spacing and height.

3.5 JOINT ROUGHNESS COEFFICIENT:

Roughness is an important controlling factor for the shear behaviour of rock joints. The expressed roughness is terms of a joint roughness coefficient that can be either determined by tilt, push or pull test on rock samples or by visual comparison with a set of roughness profile. The joint roughness coefficient (JRC) represents a sliding scale of roughness which varies from approximately 20 to 0 from the roughest to smoothest surface respectively.

3.6 SCALE EFFECT:

The strength of rock materials decreases with increase of the volume of the test specimen. This property is called scale effect which can also be observed in soft rocks. Scale effects are more pronounced in case of rough, undulating joint type where as they are virtually absent in case of planar joints. The key factor seems to be the involvement of different length of joints. The results showed that both JRC and JCS reduced to the changing stiffness of a rock mass as the block size or joint spacing increases or decrease to overcome the effects of size suggested tilt or pull tests on singly jointed naturally occurring blocks of length equal to mean joint spacing to derive almost scale free estimates of JRC as

$$JRC = (\alpha - \phi_r)/ \log (JCS - \sigma_{n0})$$

Where α = tilt angle

σ_{n0} = normal stress when sliding occurs

4. STRENGTH CRITEREON FOR ANISOTROPIC ROCKS:

Unlike isotropic rocks, the strength criterion for anisotropic rocks is more complicated because of the variation in the orientation angle β. A number of empirical strength criteria have been proposed in the past by Navier – Couloumb and Griffith.

An idealised cylindrical specimen of anisotropic rock with an oblique plane of weakness makes an angle β. The angle β is designated as the orientation angle. Hock and Brown (1980) showed clearly the strength of all rocks is maximum at β = 0º to 90º and is minimum for β = 20º to 30 º.in

Using the non linear failure envelopes predicted by classical Griffith's theory for plane compression and through a process of trial and error, Hock and Brown (1980) presented an empirical failure criterion applicable for both isotropic and anisotropic rock.

$$\sigma_1 = \sigma_3 + m\ \sigma_c.\ \sigma_3 + s.\ \sigma_c^{2})^{1/2}$$

Where s= 1 for intact rock and

= 0 for crushed rock

m varies widely as a function of rock quality and type.

Ramamurthy et al. and Rao et al. proposed an empirical strength criterion to account for the non-linear strength response of isotropic intact rocks in the following form:

$$(\sigma_1 - \sigma_3)/\ \sigma_3 = Bi\ (\sigma_{ci}/\ \sigma_3)^{\alpha i}$$

Where σ_{ci} is the uniaxial compressive strength of intact rock without a weak plane, σ_1 and σ_3 are plane stresses, αi is the slope of plot between $(\sigma_1 - \sigma_3)/\ \sigma_3$ and $(\sigma_{ci}/\ \sigma_3)$ on the log-log scale and Bi $=(\sigma_1 - \sigma_3)/\ \sigma_3$ and $(\sigma_{ci}/\ \sigma_3) = 1$, αi and Bi are considered as strength parameters. The authors had suggested a constant value of 0.8 for αi at all orientations even for intact anisotropic rocks. Owing to the fact that Bi parameter did not vary much in their analysis, a constant value for Bi as well could have assumed. The variation in the value of Bi was calculated corresponding to a constant average value of αi = 0.8. Ramamurthy et al. on the basis of the results obtained from the triaxial compressive strength on three anisotropic rocks viz. quartzite, carbonaceous and micaceous phyllites and plots between log and log for different orientations, have concluded that even for intact anisotropic rocks, the strength parameters denoted by αj and Bj cannot be taken as constants and these parameters showed systematic variation with of anisotropic rock and orientation angle refers to intact rock without weak plane and j with weak planes.

In order to predict the strength of anisotropic or jointed rock from the proposed criterion as $(\sigma_1 - \sigma_3)/\sigma_3 = Bi\,(\sigma_{ci}/\sigma_3)^{\alpha i}$.

4.1 INFLUENCE OF SINGLE PLANE OF WEAKNESS:

In a laboratory test the orientation of plane of weakness with respect to principal stress direction remains unaltered. Variation of the orientation of this plane can be achieved by obtaining cores in different directions. In a field situation either in foundations of dam around underground or open excavation the orientation of joint system remains stationary but the direction of principal stress rotates resulting in a change in the strength of rock mass.

4.2 STUDY ON PLANAR JOINTS:

In the present study, plaster of Paris specimen will be prepared to have the joint plane `at desired orientation using matching metal casting to obtain joint plane with possible limit of tolerance. For sand stone and granite, the specimen should cut along the desired direction.

TABLE4.1

PROPERTIES OF PARAMETERS ACCORDING TO THE MATERIAL

Sl no.	Property of parameter	Materials (Yaji – 1984)		
		Plaster of Paris	Sandstone	Granite
1.	Mass density (KN/ m3)	12.25	22.5	26.5
2.	Specific gravity	2.61	2.63	2.69
3.	porosity (%)	60	12	<1
4.	Uniaxial compressive strength	9.5	70	123
5.	Tensile strength	2.6	7.8	14.7
6.	Tangent modulus	1	5.1	10.8
7.	Cohesion intercept	2.17	14	25.5
8.	Angle of friction	40.5	44	46.5

4.3 INFLUENCE OF SAMPLE SIZE

It is generally assumed that there is a significant reduction in strength with increasing sample size. Based upon an analysis of published data, Hoek and Brown (1980a) have suggested that the uniaxial compressive strength σcd of a rock specimen with a diameter of d mm is related to the uniaxial compressive strength $\sigma c50$ of a 50 mm diameter sample by the following relationship:

$$\sigma_{cd} = \sigma_{c50}\,(50/d)^{\,0.18}$$

4.4 INFLUENCE OF NUMBER AND LOCATION OF JOINTS

For plaster of Paris representing weak rock, the variation of number of horizontal joints per meter length (Jn, joint frequency) with the ratio of uniaxial strength of joint and intact specimens under unconfined compression.

The location of a single joint with respect to the loading surface defined by df = Dj/B (ratio of depth of joint Dj wo the width or diameter B of the loaded area) greatly influences the strength of rock when the joint is placed very close to the loading face the strength of joint away from the loading face the strength of jointed rock mass increases and attain a value the same as that of intact rock so long as the joints within the depth equal to the width of loaded arrears. The stiffness of the rock is the highest when the joint is very close to the loading face contrary to what has been observed for strength influence of the location of a joint on the stiffness continuous to decrease even up to a depth twice the width if the loaded area.

4.5 PARAMETERS CHARECTERISING THE TYPE OF ANISOTROPY

Broadly three possible parameters define the concept of strength anisotropy in rocks. These are:

1. Location of maximum and minimum compressive strength (σ_{cj}) in the anisotropy curve in term of the orientation angle (β)
2. The value of the uniaxial compressive strength at these orientations and
3. General shape of the anisotropy curve.

5. UNIAXIAL COMPRESSIVE STRENGTH

The uniaxial compressive strength of a rock mass is represented in a non dimensional form as the ratio of the compressive strength of jointed rock to that of intact rock. The uniaxial compressive strength ratio is expressed as

$$\sigma_{cr} = \sigma_{cj} / \sigma_{ci}$$

Where σ_{cj} = uniaxial compressive strength of jointed rock and σ_{ci} uniaxial strength of intact rock. The uniaxial compressive strength ratio of the experimental data is plotted against the joint factor. The joint factor for the experimental specimens is estimated based on the joint orientation, joint strength. Based on the statistical analysis of the data, empirical relationships for the uniaxial compressive strength ratio as a function of joint factor (Jf) are derived.

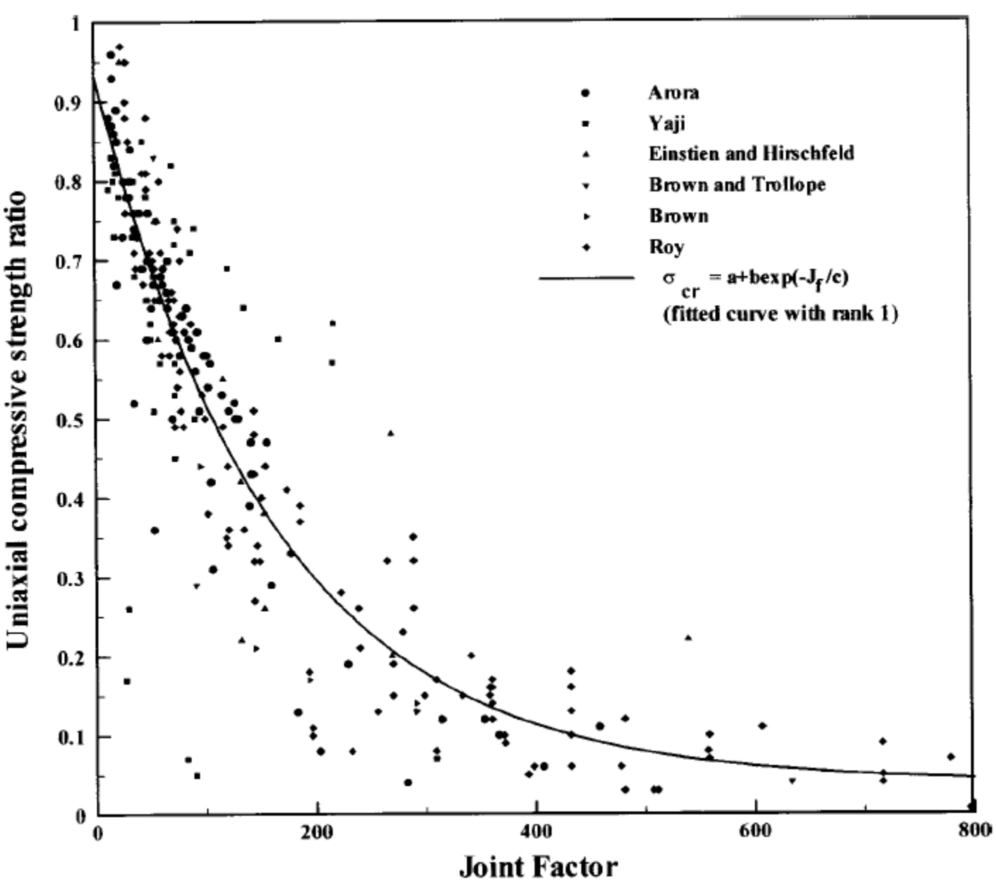


Fig - 5.1
Uniaxial compressive strength Vs. Joint factor

6. ELASTIC MODULUS

Elastic modulus expressed as tangent modulus at 50% of the failure stress is considered in this analysis. The elastic modulus ratio is expressed as:

$$Er= Ej/ Ei$$

Where Ej=tangent modulus of the jointed rock and Ei=tangent modulus of the intact rock.

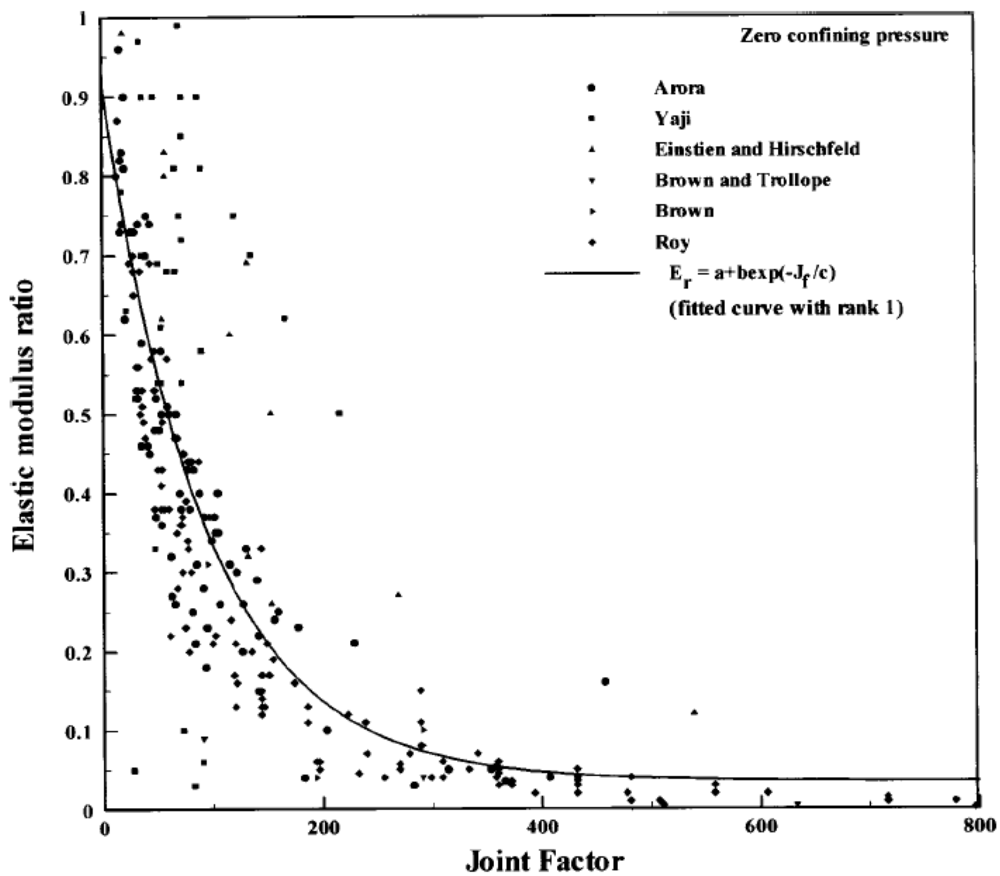


Fig 5.2
Elastic modulus Ration Vs. Joint factor

7. FAILURE MODES IN ROCK

The failure modes were identified based on the visual observations at the time of failure. The failure modes obtained are:

(i) Splitting of intact material of the elemental blocks,

(ii) Shearing of intact block material,

(iii) Rotation of the blocks, and

(iv) Sliding along the critical joints.

These modes were observed to depend on the combination of orientation h and the stepping. The angle θ in this study represents the angle between the normal to the joint plane and the loading direction, whereas the stepping represents the level/extent of interlocking of the mass. The following observations were made on the effect of the orientation of the joints and their interlocking on the failure modes. These observations may be used as rough guidelines to assess the probable modes of failure under a uniaxial loading condition in the field.

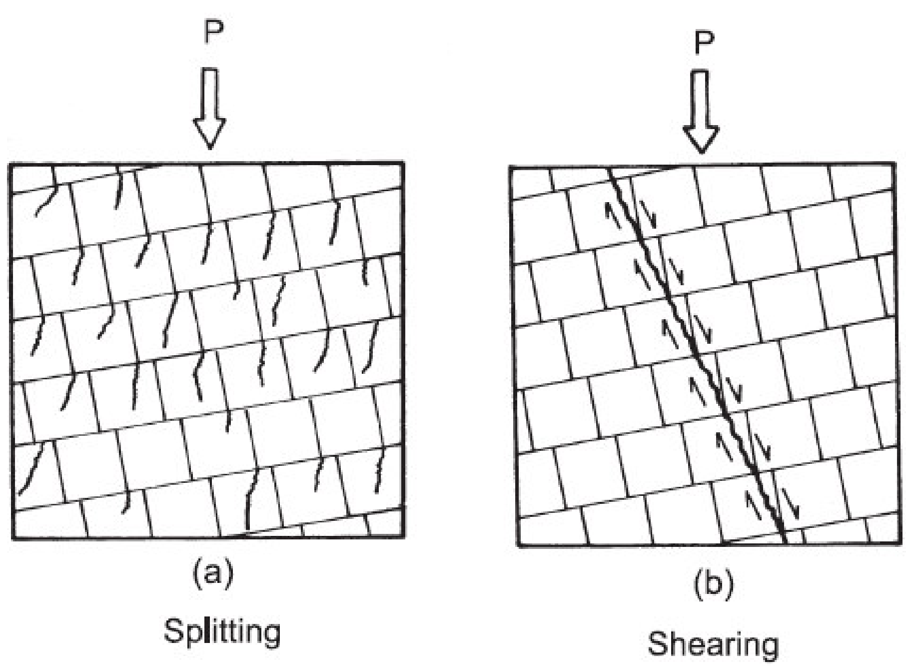


Fig – 7.1
Splitting and shearing modes of failures in rocks.

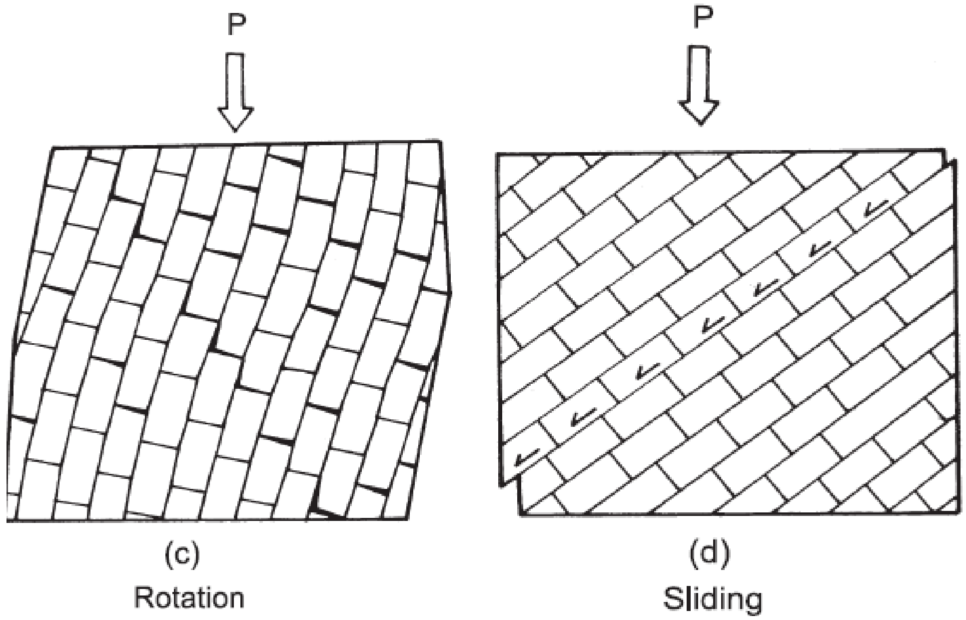


Fig – 7.2
Rotation and sliding modes of failures

7.1 SPLITTING

Material fails due to tensile stresses developed inside the elemental blocks. The cracks are roughly vertical with no sign of shearing. The specimen fails in this mode when joints are either horizontal or vertical and are tightly interlocked due to stepping.

7.2 SHEARING

In this category, the specimen fails due to shearing of the elemental block material. Failure planes are inclined and are marked with signs of displacements and formation of fractured material along the sheared zones. This failure mode occurs when the continuous joints are close to horizontal (i.e., $\theta <= 10^{\circ}$) and the mass is moderately interlocked.

As the angle h increases, the tendency to fail in shearing reduces, and sliding takes place. For $\theta \approx 30^{\circ}$, shearing occurs only if the mass is highly interlocked due to stepping.

7.3 SLIDING

The specimen fails due to sliding on the continuous joints. The mode is associated with large deformations, stick–slip phenomenon, and poorly defined peak in stress–strain curves. This mode occurs in the specimen with joints inclined between $\theta \approx 20^\circ - 30^\circ$ if the interlocking is nil or low.

For orientations, $\theta = 35^\circ - 65^\circ$ sliding occurs invariably for all the interlocking conditions.

7.4 ROTATION

The mass fails due to rotation of the elemental blocks. It occurs for all interlocking conditions if the continuous joints have $\theta > 70^\circ$, except for θ equal to 90° when splitting is the most probable failure mode.

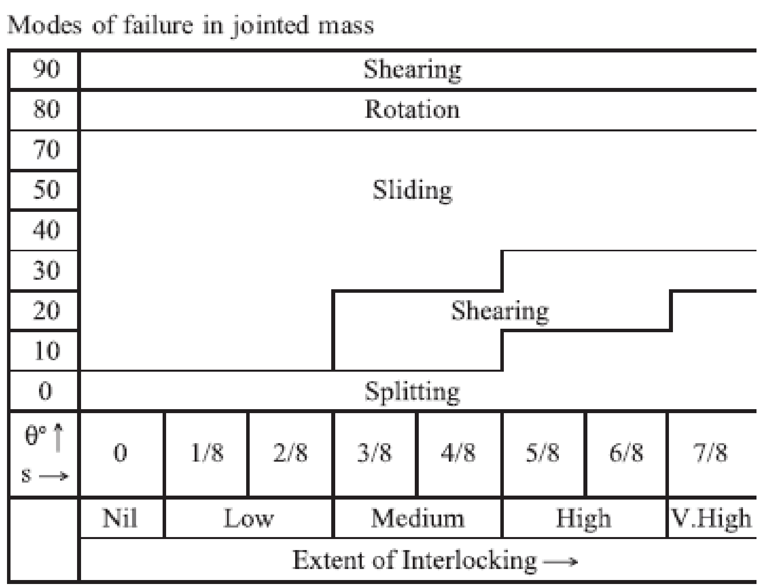


Fig- 7.3
MODES OF FAILURE IN JOINTED ROCK MASS.

Table -7.1
STRENGTH OF JOINTED AND INTACT ROCK MASS
(Ramamurthy and Arora, 1993)

Class	Description	UCS, MPa
A	Very high strength	>250
B	High strength	100–250
C	Moderate strength	50–100
D	Medium strength	25–50
E	Low strength	5–25
F	Very low strength	<5

Table – 7.2
MODULUS RATIO CLASSIFICATION OF INTACT AND JOINTED ROCKS
(Ramamurthy and Arora, 1993)

Class	Description	Modulus ratio, M_{rj}
A	Very high modulus ratio	>500
B	High modulus ratio	200–500
C	Medium modulus ratio	100–200
D	Low modulus ratio	50–100
E	Very low modulus ratio	<50

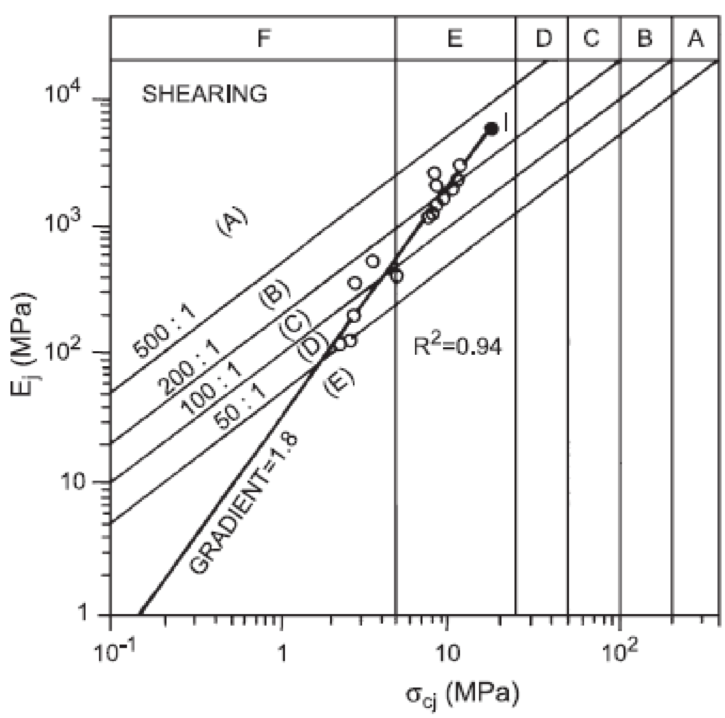


Fig – 7.3
STRENGTH AND TANGENT MODULUS VALUES FOR SHEARING MODE OF FAILURE (CHART AS PER RAMAMURTHY AND ARORA, 1993).

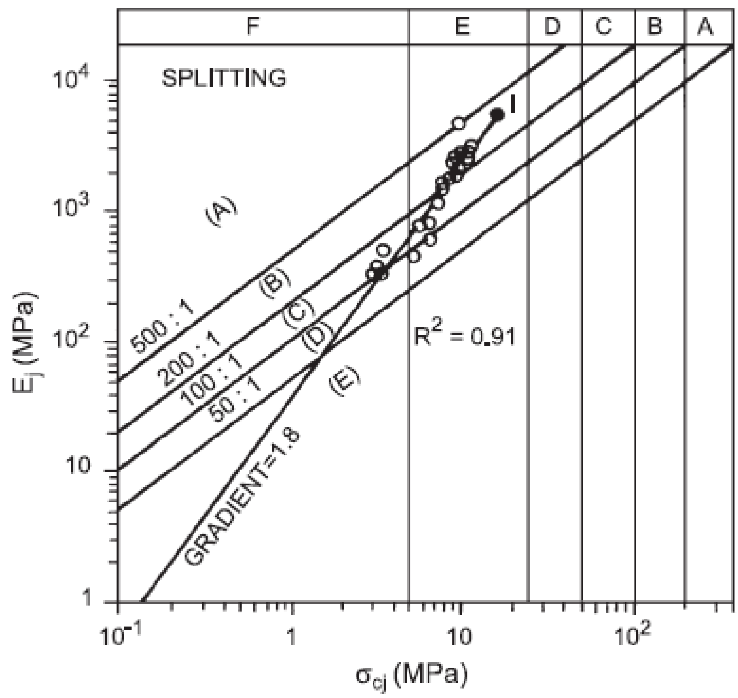


FIG – 7.4

STRENGTH AND TANGENT MODULUS VALUES FOR SPLITTING MODE OF FAILURE (CHART AS PER RAMAMURTHY AND ARORA, 1993).

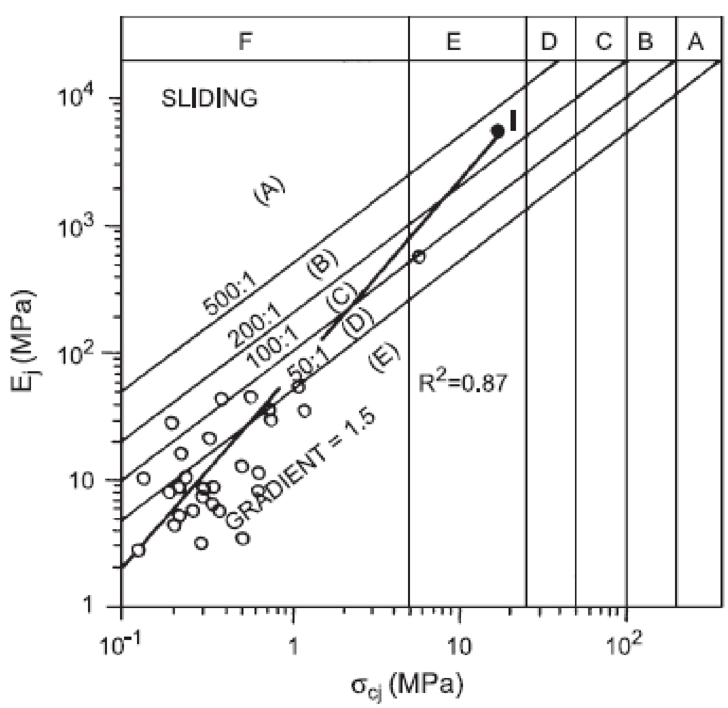


FIG – 7.5

STRENGTH AND TANGENT MODULUS VALUES FOR SLIDING MODE OF FAILURE (CHART AS PER RAMAMURTHY AND ARORA, 1993)

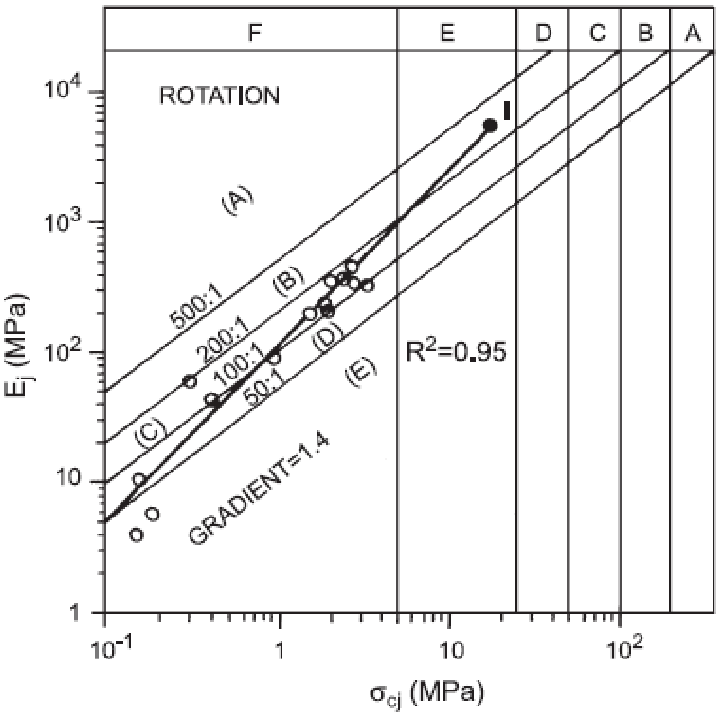


Fig – 7.6

STRENGTH AND TANGENT MODULUS VALUES FOR ROTATION MODE OF FAILURE (CHART AS PER RAMAMURTHY AND ARORA, 1993).

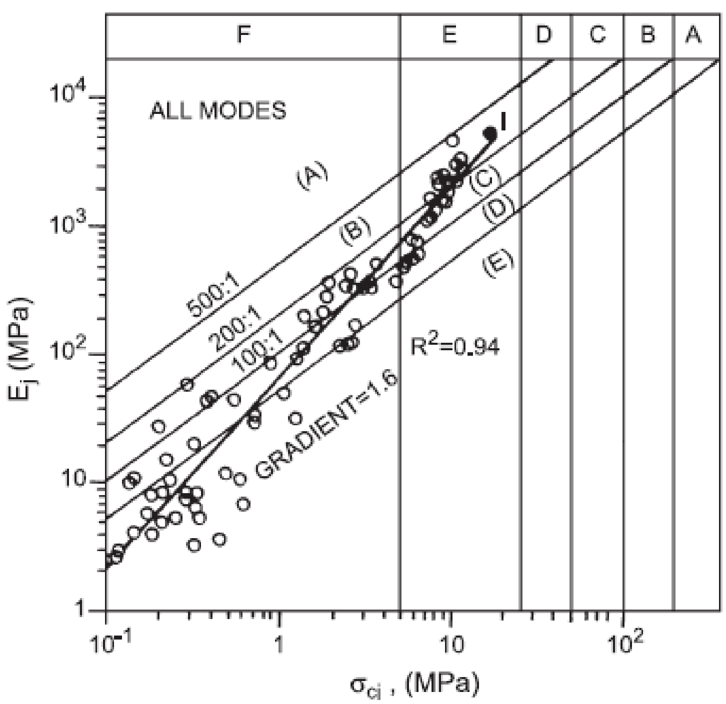


Fig – 7.7

STRENGTH AND TANGENT MODULUS VALUES FOR ALL MODES OF FAILURE (CHART AS PER RAMAMURTHY AND ARORA, 1993).

8. LABORATORY INVESTIGATION

A large number of uniaxial compressive tests are conducted on the prepared specimens of jointed block mass having various combinations of orientations and different levels of interlocking of joints for obtaining the ultimate strength of jointed rock mass. In this semester no laboratory tests were conducted. It is due for next semester.

8.1 UNIAXIAL COMPRESSIVE STRENGHT TEST:

In Uniaxial test the cylinder specimen of the soil is subjected to major principal stress till the specimen fails due to shearing along a critical plane of failure. In this test the core should be circular in shape, length 2.5 to 3 times the diameter; end shall be flat within 0.02mm. Perpendicularity of the axis shall not be deviated by 0.001radian and the specimen shall be tested within 30days. The applied load on the specimen shall be at the rate of 5.1to 10.2 kgf/cm^2/sec. The diameter of the specimen shall be either 25mm or 50mm. After measuring the load bearing surface areas the well prepared specimen is put in between the two steel plates of the testing machine and load applied at the predetermined rate along the axis of the sample till the sample fails. The deformation of the sample is measured with the help of a separate dial gauge. The ends of the cylindrical specimen are hollowed in the form of cone. The cone seatings reduce the tendency of the specimen to become barrel shaped by reducing end straits. During the test, load versus deformation readings are taken and a graph is plotted. When a brittle failure occurs, the proving ring dial indicates a definite maximum load which drops rapidly with the further increase if strain. The applied load at the point of failure should be noted. The load is divided by load borne by the bearing surface of the specimen will give the Uniaxial compressive strength of the same. Generally 7 to 10 tests are to be done for a particular rock type to establish the average values of its compressive strength. For irregular specimen more tests are recommended.

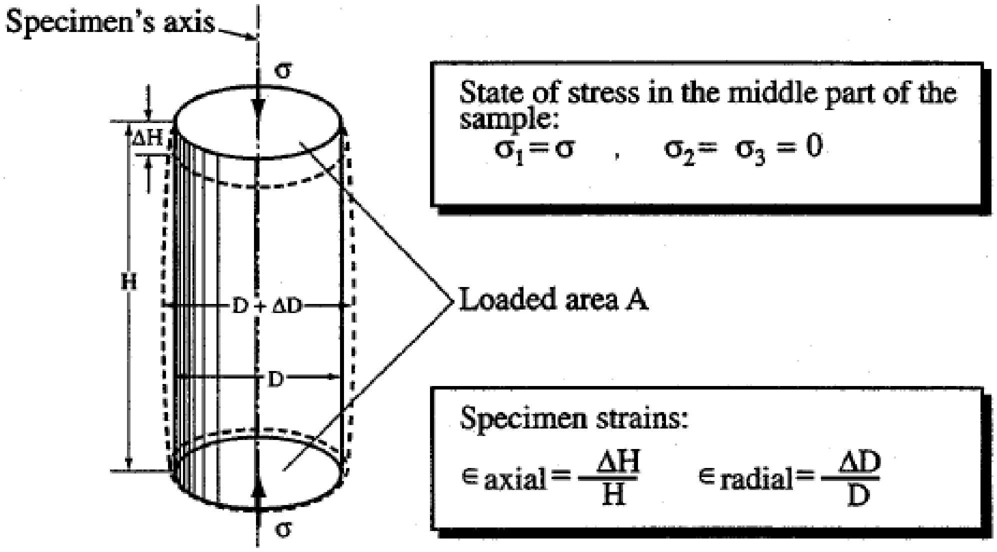


Fig – 8.1

STRESSES IN A UCS SPECIMEN

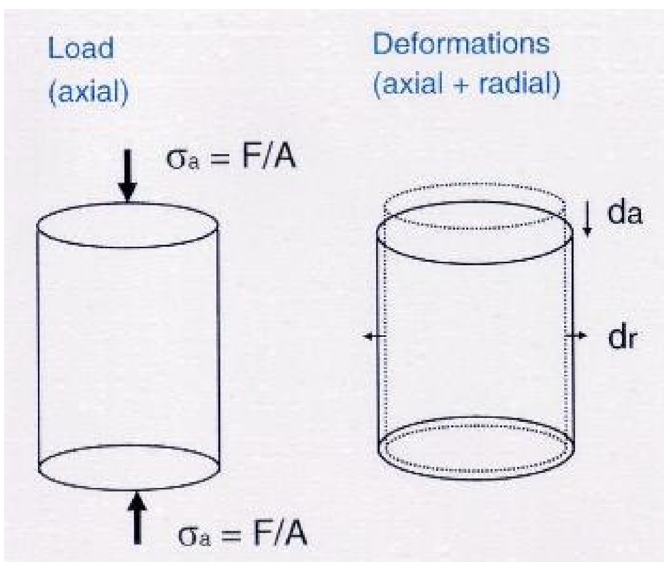


Fig – 8.2

LOADING ON THE SPECIMEN.

8.2 MATERIAL OF THE SPECIMEN:

Various materials like plaster of Paris, Kota sandstone, Jamarani, sandstone, Agra sandstone, Granite, Gypsum plaster can be used for preparing replicas of jointed rock mass in a laboratory. Research is still being conducted on getting a model material to reproduce the natural rock mass and get satisfactory results in understanding the failure mechanism and strength behaviour.

Intact and joint rock mass properties of the frequently used materials are as given below:

Table – 8.1

DIFFERENT ROCK TYPES AND THEIR PROPERTIES.

Rock type	σci (Mpa)	Ei (Mpa)	σ3 (Mpa)	Joint types	Jf	classification
Plaster of paris	9 - 12	1,000–5,500	0 - 7	Single smooth and stepped, berm shaped, multiple parallel ~2–7 joints. one to two sets of closed and gouge filled joints (β50° – 90°)	10–800	Soft rock to hard rock
Kota Sandstone	50	5,100–7,750	0–10	Single joint (β 50° – 90°)	10–100	Hard rock
Jamarani Sandstone	55.07	7,360–14,780	0–10	Single and multiple parallel joints ~2–7 joints (β 50° – 90°)	10–500	hard rock
Agra Sandstone	110	20,000 – 26,000	0–10	Single and multiple parallel joints ~2–7 joints (β 50° – 90°)	10–400	Extremely hard rock
Granite	123	10,800 – 12,800	0–10	Single joint (β 50° – 90°)	10–100	Extremely Hard rock
Gypsum plaster	20 – 50	4,000–30,000	0–14	Single and multiple joints ~parallel, Perpendicular prismatic blocks,	10–700	Medium hard rock to hard rock

				and inclined (β 50° – 90°)		

Usually plaster of Paris is used as a model material in simulation of model material to simulate the weak rock mass in the field. Because of its ease in casting, flexible behaviour, low cost, instant hardening and commercial availability Plaster of Paris is the most frequently used material in these kinds of tests. Moreover hardened plaster of Paris replicates the behaviour of soft natural rocks. In addition to the above advantages when foreign materials such as mica, sand, calcite etc are mixed to plaster of Paris its strength can be varied from low to reasonably high values. Any type of joint can be made using plaster of Paris. Thus its reduced strength and greater deformability in relation to actual rocks has made it the ideal material for modelling in Geotechnical Engineering.

8.3 PREPARATION OF SPECIMENS:

Plaster of Paris was procured from the local market. This plaster of Paris powder was produced by pulverising burnt gypsum, is dull white in colour, with a smooth feel of cement. The entire quantity for experimentation was bought in a single lot and stored air tight to avoid the entry of atmospheric moisture. This is highly essential in order to keep the different parameters such as density, moisture content, liquid limit, plastic limit etc to remain same for all the specimens that will be prepared.

For preparation of specimens, 110gms of plaster of Paris was mixed with required quantity of distilled water to form a uniform paste. This uniform paste was then poured into a cylindrical mould, which was smeared with grease/oil to avoid any kind of void formations in the specimen and for easy extrudation. The uniform paste inside the mould was kept over the vibrating machine and was allowed to vibrate for about 3 – 4 minutes. After hardening the specimen was extracted manually from the mould by using a threaded extruder and kept at room temperature for 48 hours.

8.3.1 CURING:

After keeping at room temperature for 48 hours, plaster of Paris specimens were placed inside a desiccators containing 1N sulphuric acid ($H_2 SO_4$). This was done to maintain the relative humidity in the range of 40 % - 60%. The plaster of

Paris specimens were allowed to cure in the desiccators kept at room temperature till constant weight is obtained. i.e. is 15 days in the present case.

8.3.2 INTRODUCTION OF ANISOTRPY:

In the rock mass joint planes may be oriented in different directions with respect to the stress field and this may vary from place to place. Also, during externally applied loading the principal stress direction may change. To investigate these aspects in this study, single plane of weakness and its inclination with respect to major principal stress direction should be considered.

8.3.3 DEVELOPING JOINTS IN SPECIMENS:

For developing joints in the plaster of Paris specimens, the following instruments were used –

i. Scale, pencil and protractor
ii. Chisel
iii. V block
iv. Light weight hammer

Two longitudinal lines were drawn on the specimen just opposite to each other. At the centre of the line the designated angle β was marked with the help of a protractor. This marked specimen is then placed on the V block. The chisel was aligned in line along the drawn for the desired angle β. afterwards the chisel was hammered slowly with a light weight hammer to break the specimen along the drawn line. The joints thus formed are rough joints.

Two longitudinal lines were drawn on the specimen just opposite to each other. At the centre of the line the designated angle (β) was marked with the help of protractor. Then this marked specimen was placed on the V block. Then the chisel was aligned along the line drawn for the desired angle (β). Afterwards, the chisel was hammered slowly with a light weight hammer to break the specimen along the line drawn.

A joint so formed will be rough in nature and hence comes under the category of rough joints.

8.4 EXPERIMENTAL SET UP AND PROCEDURE:

Uniaxial compression tests and direct shear test were done on the prepared specimens to obtain their uniaxial compressive strength, deformation modulus and shear parameters. These laboratory tests were done according to I.S.R.M. and I.S. codes.

The direct shear tests were conducted to determine the roughness factor, joint strength, r (= tanφ j) in order to evaluate the joint factor, Jf, (Arora 1987). These tests were carried out in the conventional direct shear apparatus (IS: 1129, 1985) with certain modifications required for placement of specimens inside the box. Two identical wooden cubical blocks of size 59 x 59 x 12 mm each and having a circular hole of diameter 39 mm at the centre were inserted into two halves of the shear box (60 x 60 mm). The cylindrical specimen broken into the two equal parts was fitted into the circular hole of the wooden blocks, so that the broken surface match together and laid on the place of shear i.e. the contact surface of two halves of the shear box.

The uniaxial compression tests were conducted on the conventional strain controlled machine at a strain rate of 0.5 mm/ minute (0.02 mm/min).

8.5 PARAMETERS STUDIED:

The main objective of the experimental investigation is to study the following aspects:

1) The effect of joint factor (Jf) on the strength characteristics of single jointed and double jointed specimens.

2) The shear strength behaviour of plaster of Paris.

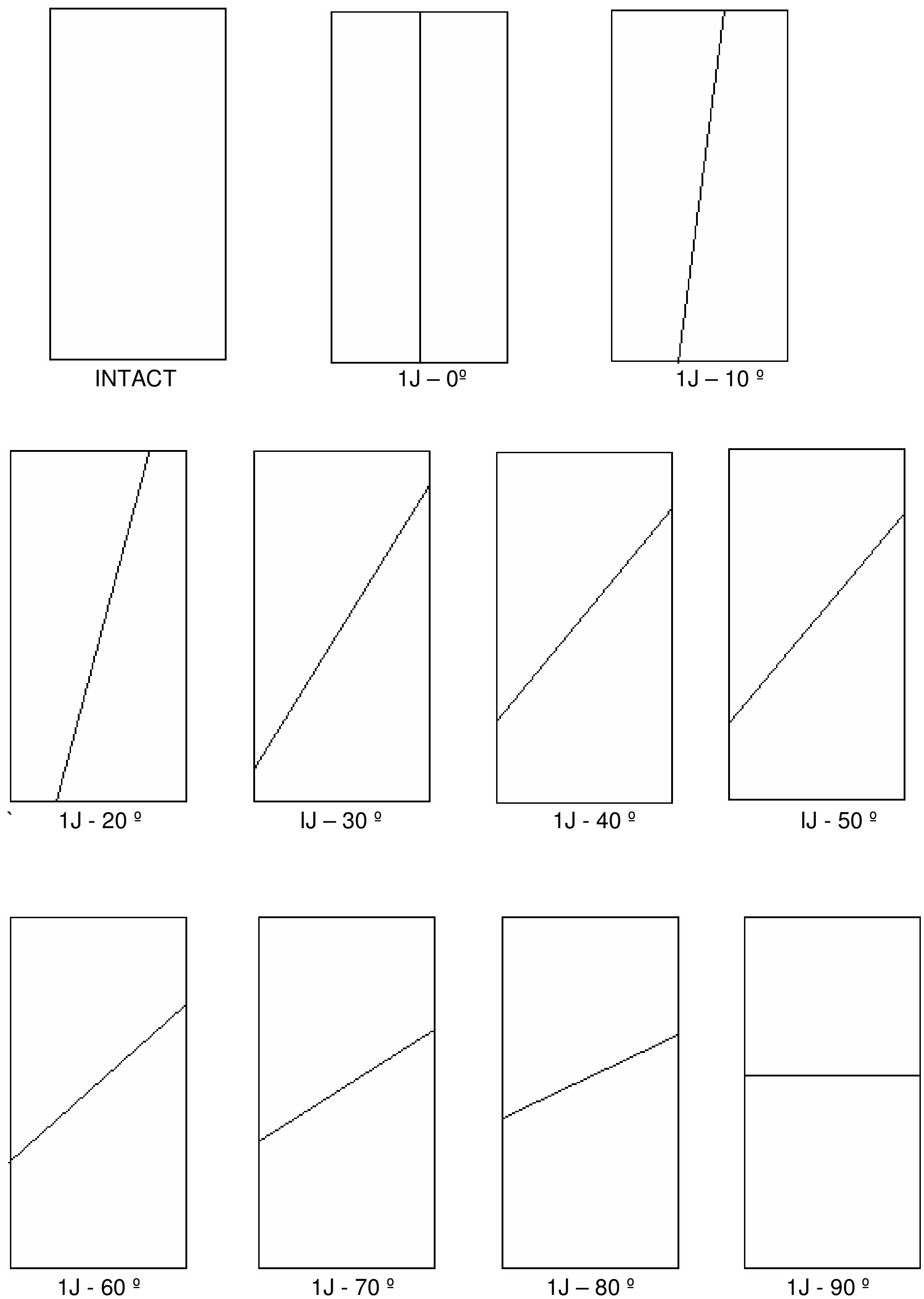
3) Variation of modulus ratio for intact and jointed specimens.

In view of the above, the experiments have been conducted and the different parameters evaluated are given below.

Uniaxial compression test were done for single joint at various inclinations i.e. β = 0°, 10 °, 20 °, 30 , 40 °, 50 °, 60°, 70 °, 80 °, 90 ° and double joints at

β = 60-60, 70-70, 80-80, 90-90.

For each orientation of joints, three U.C.S tests were conducted as shown in the table. These are shown in the figures. The joined specimens were placed inside a rubber membrane before testing of U.C.S to avoid slippage along the joints just after application of the load.



TYPES OF JOINTS STUDIED IN PLASTER OF PARIS SPECIMENS.

9. RESULTS AND DISCUSSIONS:

9.1 ROUGHNESS PARAMETER:

The roughness parameter (r) which is the tangent value of the friction angle (φj) was obtained from the direct shear test conducted at different normal stresses. The variation of shear stress with normal stress for specimens tested in direct shear tests are illustrated in the fig.10.1 and their corresponding values are given in the table 10.1. The values of cohesion (Cj) for jointed specimens of plaster of Paris has been found to be 0.19 Mpa and values of friction angle (φ) found to be 40º. Hence the roughness parameter ($r = \tan\varphi_j$) comes to be 0.861 for the specimens of plaster of Paris tested.

9.2. UNIAXIAL COMPRESSION TEST RESULTS:

9.2.1 FOR INTACT SPECIMENS:

The variations of the stress with strain as obtained in uniaxial compression test for the intact specimen of plaster of Paris is illustrated in the fig 10.2 and its corresponding stress Vs strain values are presented in table 10.2. The value of uniaxial compression strength (σ_{ci}) evaluated from the above tests was found to be Mpa. The modulus of elasticity of intact specimen (M_n) was calculated at 50%.

TABLE 9.1

VALUES OF SHEAR STRESS FOR DIFFERENT VALUES OF NORMAL STRESS ON JOINTED SPECIMENS OF PLASTER OF PARIS IN DIRECT SHEAR STRESS TEST.

CROSS SECTIONAL AREA OF SAMPLES = 1134 mm^2

NORMAL STRESS, σn (Mpa)	SHEAR STRESS, ح (Mpa)
0.08	0.290
0.16	0.410
0.24	0.426

0.32	0.497
0.40	0.579

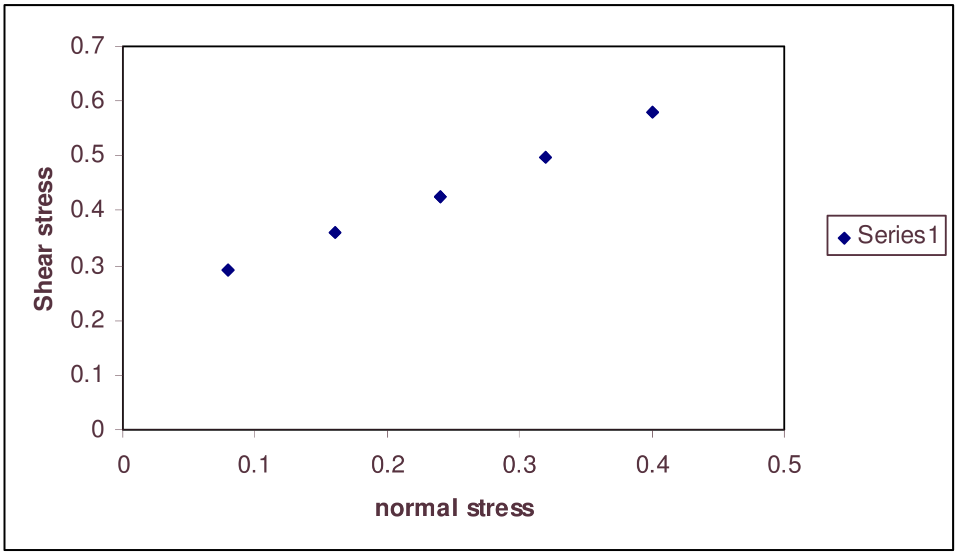


Fig.9.1

NORMAL STRESS Vs SHEAR STRESS

Length of specimen = 76mm

Diameter of specimen = 38mm

Cross sectional area of the specimen = 1134mm^2

TABLE 9.2
VALUES OF STRESS AND STRAIN FOR INTACT SPECIMENS:

Axial strain, $\epsilon a(\%)$	Uniaxial compressive strength, σci (Mpa)
0	0
0.560	4.563

0.697	6.944
1.403	9.211
2.130	8.919
2.693	7.977

The modulus of elasticity for intact specimen (E_{ti}) has been calculated at 50% of σ_{ci} value to account the tangent modulus. The value of Mn for the intact specimens was found to be 0.9211 x 10^3 Mpa.

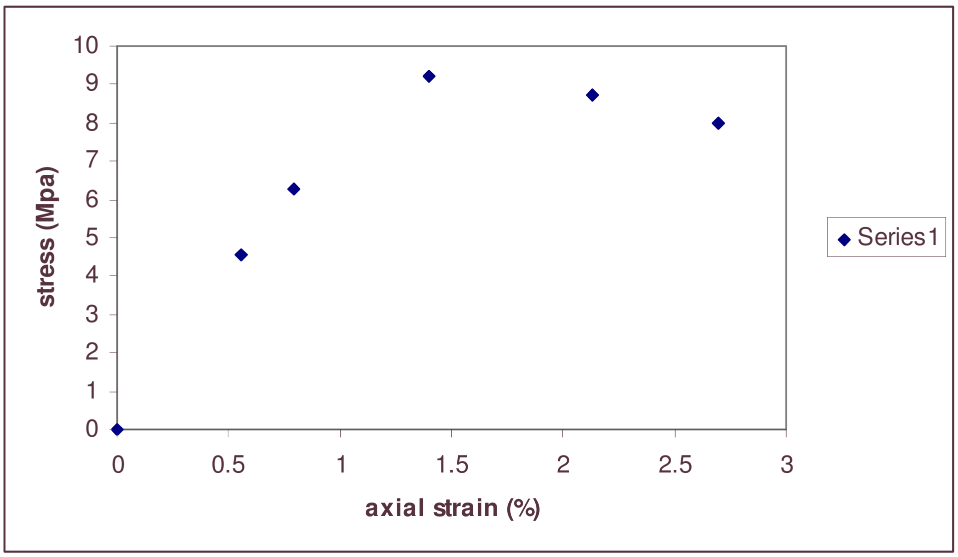


Fig - 9.2 AXIAL STRAIN /Vs STRESS

Table 9.3

PHYSICAL AND ENGINNERING PROPERTIES OF PLASTER OF PARIS USED FOR JOINTS STUDIED:

Sl No.	Property/Parameter	Values
1	Mass density (KN/m3)	13.94

2	Specific gravity	2.81
3	Uniaxial compressive strength, σci (Mpa)	9.25
4	Tangent modulus, (E_{ti}) (Mpa)	0.921
5	Cohesion intercept, Cj (Mpa)	0.19
6	Angle of friction, φj (degree)	40º

FOR INTACT/JOINTED SPECIMENS:

9.2.2.1 STRENGTH CRITERIA

The uniaxial compressive strength of intact specimens obtained from the test results has already been discussed in 10.2.1. In similar manner the uniaxial compressive strength (σ_{ci}) as well as modulus of elasticity (E_{ti}) for the intact jointed specimens was evaluated after testing the jointed specimens. In this case, the close/intact jointed specimens are placed inside a rubber membrane before testing, to avoid slippage along the critical joints.

After obtaining the values of (σ_{ci}) and E_{ti} for different orientations (β) of joints it was observed that the intact jointed specimens exhibit minimum strength when the joint orientation angle was at 30º. The values of (σ_{cr}) for different orientation angle (β) were obtained with the help of following relationship:

$$(\sigma_{cr}) = (\sigma_{cj}) / (\sigma_{ci})$$

The values of joint factor (J_f) were evaluated by using the relationship:

$$J_f = J_n / (nxr)$$

TABLE 9.4

VALUES OF Jn, Jf, σ_{cj}, σ_{cr} FOR INTACT AND JOINTED SPECIMENS

Joint type in degree	J_n	n	R (= tanφ j)	J_f = Jn/(nxr)	(σ_{cj}) (Mpa)	(σ_{cr}) = (σ_{cj}/σ_{ci})
0	13	0.810	0.856	18.749	7.419	0.802
10	13	0.460	0.856	33.02	6.592	0.713
20	13	0.105	0.856	144.63	2.98	0.321
30	13	0.046	0.856	330.15	0.473	0.051
40	13	0.071	0.856	213.9	2.126	0.229
50	13	0.306	0.856	49.63	4.937	0.534
60	13	0.465	0.856	32.66	6.181	0.668
70	13	0.634	0.856	23.95	6.723	0.727
80	13	0.814	0.856	18.657	7.658	0.828
90	13	1.00	0.856	15.187	8.977	0.970

DOUBLE JOINT

Joint type (in degree)	Jn	n	r (= tanφ j)	J_f = Jn/(nxr)	(σ_{cj}) (Mpa)	(σ_{cr}) = (σ_{cj}/σ_{ci})
60-60	26	0.46	0.856	48.380	3.455	0.373
70-70	26	0.64	0.856	34.775	5.781	0.624
80-80	26	0.82	0.856	27.141	6.637	0.717
90-90	26	0.95	0.856	23.427	7.895	0.853

(σ_{ci} = 9.25 Mpa)

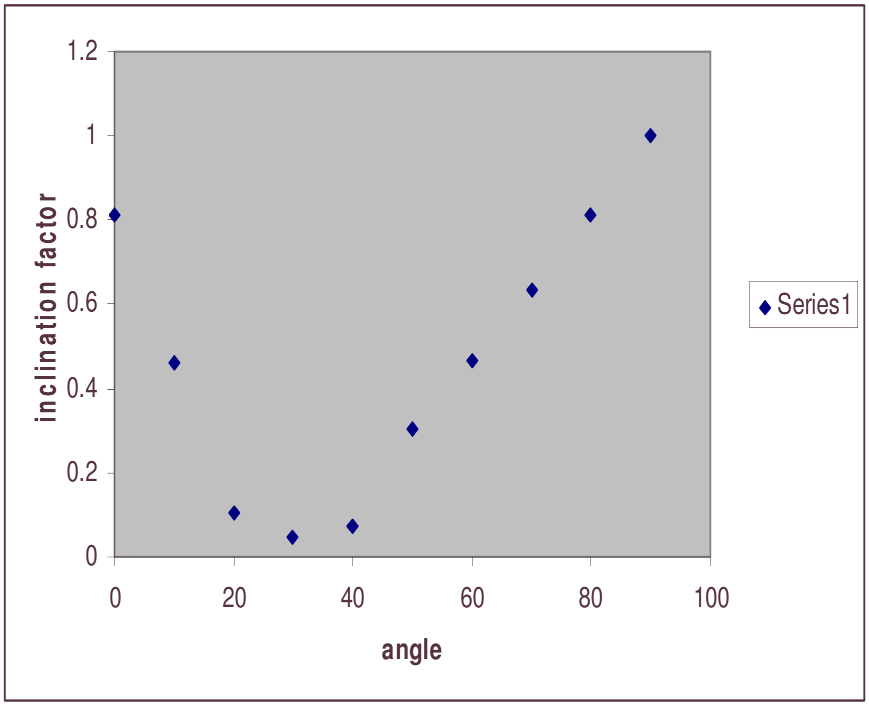


Fig. 9.3
ANGLE (β) Vs INCLINATION FACTOR

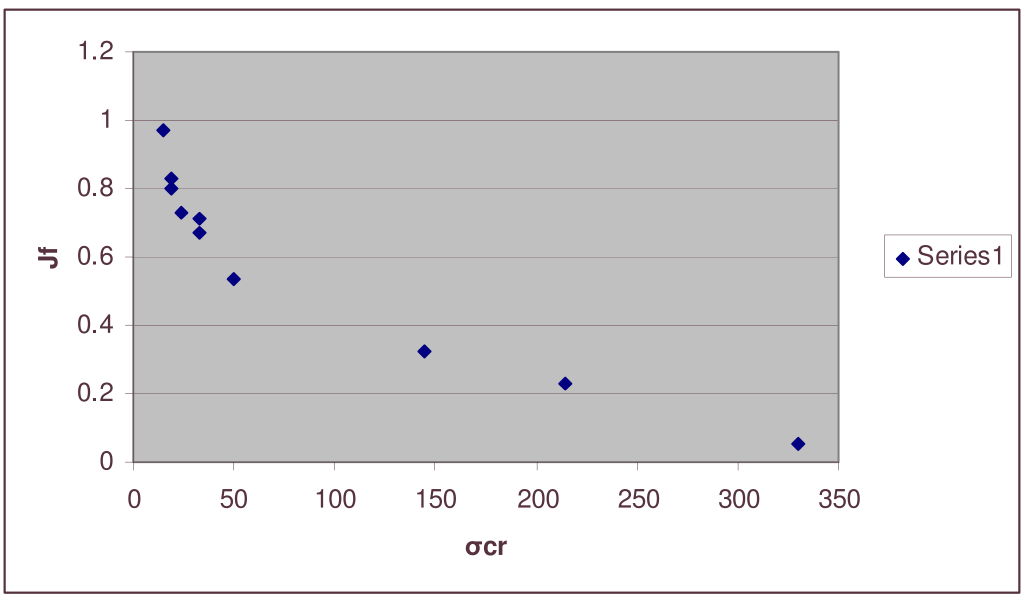


FIG. 9.4
(σ_{cr}) Vs JOINT FACTOR

The values of modulus of elasticity (Et) for different joint orientation angles for jointed specimens have been evaluated from the curves of stress Vs strain at 50% of corresponding σ_{cj} values to account for the tangent modulus.

TABLE 9.5
VALUES OF Mrj, Mr FOR INTACT JOINTED SPECIMENS

Joint type (in degree)	Jn	n	r (= tan φj)	Erj (Mpa)	Er = Erj/Eri
0	13	0.82	0.856	910.61	0.988
10	13	0.46	0.856	801.24	0.869
20	13	0.11	0.856	644.98	0.700
30	13	0.05	0.856	230.73	0.250
40	13	0.07	0.856	600.12	0.651
50	13	0.30	0.856	785.45	0.852
60	13	0.46	0.856	802.31	0.871
70	13	0.64	0.856	850.46	0.923
80	13	0.82	0.856	889.56	0.965
90	13	0.95	0.856	915.18	0.987

Based on the results of uniaxial and triaxial testsof intact and jointed specimens conducted by Yaji (1984) and Arora(1987) the following empirical relations have been given by Arora and Ramamurthy (1984) for uniaxial compressive strength ratio ((σ_{cr}) and elastic modulus ratio (Er) of jointed rock masses.

$$(\sigma_{cr}) = (\sigma_{cj})/\ (\sigma_{ci}) = \exp\ (-0.008Jf)$$

$$Er = Ej/Ei = \exp\ (-0.0115\ ^{-2}Jf) \text{ for zero confining pressure.}$$

From the above expressions a comparative study can be made between the current experimental analysis and the empirical formulae given by Arora and Ramamurthy.

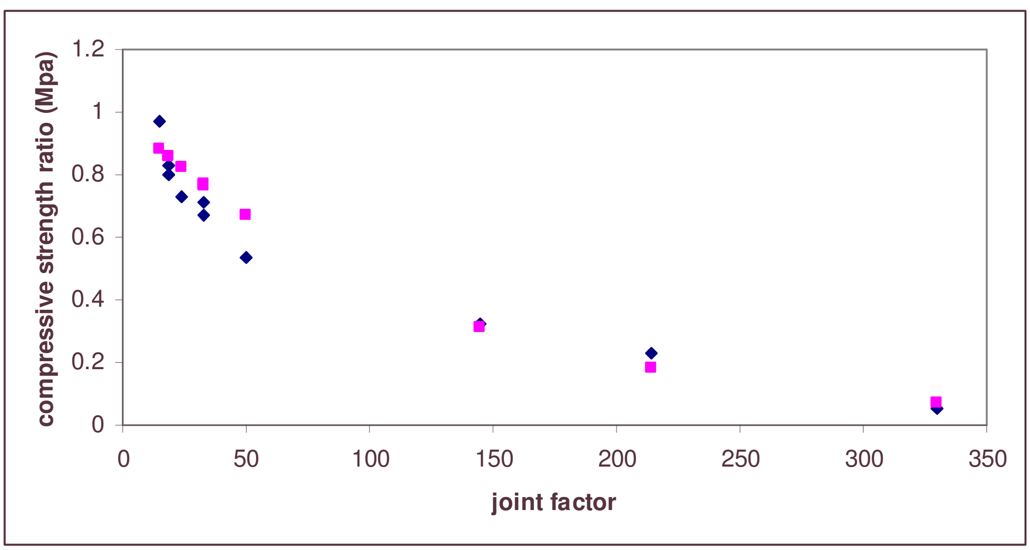


Fig – 9.5 Joint factor Vs compressive strength ratio

10. CONCLUSION

On the basis of current experimental study on the intact and jointed specimen of plaster of Paris the following conclusions are drawn:

1. The uniaxial compressive strength of intact specimen of plaster of Paris is found to be 9.25Mpa.

2. The strength of jointed specimen depends on the joint orientation β with respect to the direction of major principal stress. The strength at $\beta = 30^{\circ}$ is found to be minimum and the strength at $\beta = 90^{\circ}$ is found to be maximum.

3. As the number of joints increase the uniaxial compressive strength decreases.

4. The compressive strength is more when the double joints are made at angle of orientation at 60º – 60º to than at 90 º - 90º.

5. The values of Modulus ratio (Er) also depends on the joint orientation β. The modulus ratio is least at 30º.

6. There is not much variation between the present experimental results and those obtained from the empirical formula given by Arora and Ramamurthy.

SCOPE OF FUTURE WORK:

1. The effect of temperature, confining pressure and rate of loading on the strength characteristics can be studied.

2. Studies can be made by introducing multiple joints in varying orientation.

3. Strength and deformation behaviour of jointed specimens can be studied under triaxial conditions.

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